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Discussion of

"NEW YORK'S EXTENSION OF ITS SOURCES TO THE DELAWARE"

by Karl R. Kennison

(Proc. Paper 454)

KARL R. KENNISON,[1] M. ASCE.—Mr. Greeley's discussion dealt largely with the question of Pennsylvania's interpretation of the provision of the Supreme Court's decree of June 7, 1954, which made it clear that:

> "No diversion herein allowed shall constitute a prior appropriation of the waters of the Delaware River or confer any superiority of right upon any party hereto in respect of the use of those waters. Nothing contained in this decree shall be deemed to constitute an apportionment of the waters of the Delaware River among the parties hereto."

This is a quite unnecessary subject of discussion because it is merely a reiteration of the law in the case which was established by the original decree of 1931. The press release of June 22, 1953, which the writer quoted, was a summary of the opinions up to that date of the various parties, including Pennsylvania. Opinions expressed therein, as well as those in the other documents referred to by Mr. Greeley, were fully considered in, and finally superseded by, the report of Special Master Pantzer and the Courts' decree. The matter may be briefly summarized by quoting from a formal opinion of the Corporation Counsel of New York City which was given to Mayor Wagner March 9th, in response to his request for a ruling with reference to the appropriation by the City of money to carry out the project. The Corporation Counsel quoted the Special Master as finding upon uncontroverted evidence that:

> "in the course of a calendar year the Delaware River and its tributaries produce more than enough water to meet the present additional requirements of the City of New York, as well as the present requirements of the citizens of New Jersey, Pennsylvania and Delaware, if the surplus waters of winter and spring are stored for later release."

He also quoted the testimony of experts for New Jersey and Pennsylvania, as well as that of the writer for New York, to further reinforce his conclusions as to the adequacy of the Delaware River if properly developed to supply the needs of all the lower riparian states.

He further stated that:

> "It is clear from the record that, if New Jersey and Pennsylvania maintain the same relative use of storage for river regulation, New York's plan of development in its watershed will continue to be of great benefit, and not a detriment, to the river."

and he concluded by stating:

> "It is my opinion and I so advise you that the Board of Estimate is

1. Chf. Engr., New York Board of Water Supply, New York, N. Y.

justified upon the record and the Supreme Court decree in making the necessary appropriations for the construction of the third stage of the Delaware River project."

Discussion of
"INCINERATION AND ALTERNATIVE REFUSE DISPOSAL PROCESSES"

by Ralph Stone and F. R. Bowerman
(Proc. Paper 471)

RALPH STONE,[1] and F. R. BOWERMAN,[2] Associate Members, ASCE.—The authors very much appreciate the comprehensive and valuable discussions presented. The subject matter as described by each reviewer will be discussed and material common to the several discussions will be amplified.

Mr. George Thompson raises the point as to the influence of backyard and alley collection in creating larger quantities of refuse than does curb placement. The information for the authors' statement was developed from data collected by the Sanitary Engineering Research Project, University of California at Berkeley.[3] The principal reason that larger quantities are collected from backyard pickup is believed to be the additional convenience of backyard and side-of-the-house pickup. That is, such refuse as garden trimmings and leaves are often burned by the individual householder or paper contributed to salvage paper drives when the householder is required to haul the wastes to a location at the curb in front of the residence.

Another question concerns the importance of compacted refuse in regard to the efficiency of burning at a municipal incinerator. It is noted that at incinerators where wet paper, leaves and other combustibles have been compacted that they form a blanket on the surface fire, thereby inhibiting the combustion within the primary burning chamber. Mr. Thompson correctly implies that when refuse is processed or "fluffed" by special handling prior to burning in an incinerator that the influence of compaction is less important.

California experience with the feeding of municipal garbage to hogs indicates that it is economical and practical to sterilize garbage with steam so as to prepare a sanitary product which will not transmit diseases to hogs. Recent tests performed in California indicate that the presence of citrus peels in boiled garbage does not destroy its value for hog feeding, partly because the use of raw citrus produce has declined while the market for frozen and canned juices has developed. In any event, California citrus wastes do not now interfere with the sterilization of garbage for hog feeding. Garbage cooking, in fact, has increased the value of the waste for feed by breaking down the food products and making available additional nutrients. Garbage cooking is now similarly utilized in many other locations. With respect to garbage, Mr. Thompson presents a good yardstick for incinerator operation; namely, that an efficient plant will take any "rubbish, combustible and non-combustible, together with garbage, wrapped or raw, and burn it economically without nuisance."

Mr. William A. Xanten has emphasized the significance of fly ash control in the design of and operation of an incinerator. Fly ash control and the

1. Cons. Engr., Los Angeles, Calif.
2. Div. Engr., Los Angeles County Sanitation Districts, Los Angeles, Calif.
3. Technical Bulletin No. 8, titled "An Analysis of Refuse Collection and Sanitary Landfill Disposal," dated December, 1952.

discharge of noxious gases from the combustion system are highly technical subjects. Admittedly, one of the major disadvantages of incineration is that of discharging materials which pollute the atmosphere. In cities concerned with air pollution problems, it appears to be good policy to perhaps utilize other refuse disposal systems which do not contribute to the burden of the atmosphere. In this respect, composting, grinding of combustible rubbish to sewers, and sanitary landfill are obvious alternative systems.

Experience with fly ash control indicates that well-designed incinerators, similar in engineering characteristics to Mr. Xanten's description, reduce fly ash and unburned gaseous emissions to a level of tolerance satisfactory to many communities. In particular, the use of fog nozzles to saturate a chamber wherein combustion products are treated appears to be effective in fly ash and gas cleaning systems. The cooling effect of the fine spray condenses out particles and allows for settling of large fly ash products. A fog spray allows for greater interception of the ash particles and should be more efficient than conventional water sprays.

Particular attention should be directed to studies made for the City of New York at Battelle Memorial Institute, Columbus, Ohio, on the matter of fly ash removal. By use of a scale model, internal baffles were designed to create "dead-spots" within the chambers and breeching; such baffles have succeeded in reducing fly ash discharge to near the permissible limits. Further study along these lines may yield designs which will eliminate much of the present conflict between incinerators and their environs. Of additional interest is the new incinerator at Milwaukee which utilizes combinations of water spray, impingement baffles, and water pond to reduce fly ash discharge to acceptable levels. Unfortunately, not enough basic research in fly ash control is being conducted. Mr. Xanten's additional information concerning the design of incinerator installations is a valuable contribution to our knowledge.

Mr. Samuel A. Greeley has presented interesting data concerning economic and technical problems involved in incineration. The authors have observed several instances where two-level topography was used for plant construction thus resulting in smaller less expensive buildings and ramps. Two-level topography is of greater importance in direct-dump incinerators than in crane and bin plants; however, where two-level construction can be used, the operating expenses should be less in a crane and bin plant because a lower lift is required from the bin to the charging inlet of the incinerator.

The salvage of waste paper is an important industry in the United States and reduces the quantity of combustible rubbish subject to disposal. Although little revenue may be obtained from such salvage operation, the reduction in quantity of combustibles to be burned should provide significant savings to an operating installation.

Construction costs can be reduced at locations where a minimum protection superstructure need be built. In localities where an attractive building is not required, such as in an industrial area, and in temperate climates, such as prevail in the southwestern states of the United States, it is not necessary to use expensive enclosures. In such locations, it is possible to construct incinerators at considerably less cost than in high-class residential areas where a noise-proof and dustless type with a high degree of air pollution control is needed. It should be noted that the operating cost data presented by Mr. Greeley is approximately the same as that presented by the authors, if Mr. Greeley's estimate of $3 to $4 per ton of refuse burned (based on a capital cost of $3,000 to $4,000 per ton of rated capacity) is compared to the authors' lower capital cost estimate of $2,000 per ton. Although some direct-dump incinerators are still being built for $2,000 per rated ton,

the authors agree that today's market for crane and bin plants with air pollution control devices such as spray chambers is in the range of $3,000 to $4,000 as suggested by Mr. Greeley.

The thermodynamic information presented by Mr. Greeley is valuable and representative of many eastern incinerator plants. In many parts of the western United States gas and fuel oil is largely employed for heating of houses. As a result, a typical refuse is largely cellulosic in composition and contains little ash. Also, because of the dispersed residential pattern of living in the California area, much of combustible rubbish is of plant origin; to wit, garden trimmings, grass, leaves, etc. This explains the differences in the characteristics of hypothetical examples of refuse as described by the authors, and that presented by Mr. Greeley.

In general, the discussers seem to be in agreement with the authors on the following significant points:

1) It is most important that each municipality make careful engineering study to determine the nature and extent of local refuse disposal problems and to arrive at an engineering plan for an economical waste handling system.
2) The cost of incineration is high and economy requires careful operation and well engineered plant design.
3) Air pollution in connection with incineration is an important deterrent to incineration use. Further development of incinerator design, with particular reference to fly ash and gaseous emissions, will have to be made to insure universally acceptable incinerator application.

The authors are indebted to the reviewers for presenting valuable contributions to the store-house of refuse disposal knowledge.

In conclusion, it may be of interest to compare the economics of alternative refuse disposal processes since costs are pertinent to the selection of refuse disposal systems. While such cost data is incomplete, the following information is based on considerable experience and actual cost studies. The future development of the several refuse disposal processes directly depends upon improvements to reduce cost as well as to afford greater efficiency and sanitation.

Refuse Disposal Process	Capital Costs ($/ton of capacity/day)	Total Operating Costs Including Amortization of Capital Cost ($/ton handles/day)
Incineration	$1500 - $6000*	$2.00 - $6.00*
Burning dump	Unsatisfactory for sanitary and aesthetic reasons	
Composting	$100 - $5000*	$2.00 - $30.00
Hog feeding	$300 - $1000	Profit - $1.00*
Garbage grinding	$1000 - $2000	$2.00 - $6.00*
Reduction of Garbage	$1000 - $5000	Profit - $5.00
Salvage	0 - $1000	Profit - $1.00*
Sanitary landfill	$250 - $1000	$0.40 $1.50
Water disposal	Unsatisfactory for sanitary and aesthetic reasons	

*Higher costs are usually for small installations or for plants constructed to operate with a minimum of nuisance.

Discussion of
" FUNDAMENTAL CONCEPTS OF RECTANGULAR SETTLING TANKS "

by Alfred C. Ingersoll, Jack E. McKee, and Norman H. Brooks
(Proc. Paper 590)

H. L. THACKWELL,[1] M. ASCE.—The heading of this paper deals with rectangular settling tanks; the paper, however, deals only with rectangular tanks having horizontal flow. Upward flow rectangular tanks are numerous enough to have been mentioned; consequently, the inference drawn is that only horizontal flow tanks are to be considered.

The synopsis indicates that settling tanks should be designed on the basis of surface area and that rectangular settling tanks should be long and narrow.

[2]" Data for surface settling rates versus BOD and SS removal were plotted in a form analogous to Figures 69 to 72. A great dispersion was observed, and little correlation between surface settling rate and removal efficiency could be detected. It was inferred that surface settling rate was inferior to detention period as a measure of performance in settling tanks at military plants. This might not have been the case if flows per unit volume of tank had been larger."

For any rectangular tank having vertical sides and a uniform depth, the detention time is directly proportional to the reciprocal of the surface area. In plotting a curve having percentage of suspended solids removal as ordinates and the reciprocal of the surface area as abscissae, the shape of the curve should take the general form of the usual curve plotting per cent removal against detention time. A break in this general curve as typified in Figure 69 usually occurs between two and three hours detention time.

Reference is made again to 2, page 876, where the following quotation is noted: " It is interesting to observe that with rectangular final tanks compliance with the foregoing criteria with respect to detention period, depth, displacement velocity, and overflow rate result in a fixed design admitting of no variation. The following equation may be derived from the foregoing relationships between variables:

$$b = q/(v\ max\ d)$$

$$L = v\ max\ d/r$$

Example:

$$\text{If } q = 1.0 \text{ mgd} = 92.8 \text{ cfm}$$

$$v\ max = 0.50 \text{ f/m}$$

$$t = 2.5 \text{ hrs} = 150 \text{ min}$$

$$r = 800 \text{ gpd/sf} \doteq 0.074 \text{ fpm}$$

$$d = 9 \text{ ft}$$

1. Cons. San. Engr., Walnut Creek, Calif.
2. A quotation from Sewage Works Journal, Sept. 1946, page 883.

then

$$b = 92.8/(0.50)(9) = 20.7 \text{ ft}$$

$$L = (0.50)(9)/(0.074) = 60.5 \text{ ft}$$

In the above example, the length to width ratio is 2.93 and the length to depth ratio is 6.7. This tank would prove economical for straight line sludge collectors.

In the author's statement II Resume of Fundamental Concepts: "In a quiescent suspension, the concentration may be weak in the top layers, but it will become progressively more concentrated with increasing depth from the surface." This statement is universally true and is also true in most cases of turbid moving water, such as the Mississippi River below New Orleans. The writer made an investigation of turbidity on the river surface and 20 feet below at numerous stations and found that it was always greater at the lower depths. The cleanest water is on the surface and close to shore. The paper further states: "In flocculent suspensions, two or more particles may coalesce to form a larger agglomerate which will probably settle at an increased rate." Actually, this is an argument for deeper tanks. The following extract is also in agreement: [3]"As explained, the different settling rates of the suspended particles entering the tank cause flocculation, and effectiveness of this would be increased by greater depth in tank. The section is similar to that of raindrops on a window pane. The particle, or drops, initially move downwards very slowly, but as they fall they join together to form larger drops which move more rapidly and collect with them as they go other smaller and slower drops." This further substantiates that deeper tanks have some points of merit. Numerous mention of higher density of sludge in deep tanks over that in shallow tanks has been made by observers. The dispersion index is less in shallow tanks than in deep ones, and as stated by Phelps, [4] "The more important factors which appear to govern dispersion are shape of tank, location and type of inlet and outlet devices, temperature as affecting stratification, wind action, and velocity of flow through the tank. Long narrow tanks with maximum distance between inlet and outlet are favorable to low indices. Shallow rather than deep tanks favor good distribution of flow and minimize stratification." The above statement in general agrees with that of the authors in 590-3. Again a quotation from the authors, "There appears to be little or no correlation in actual operating records between removal efficiencies and overflow rates or detention periods. Basins that are poorly designed on the basis of theory sometimes appear to perform better than well-designed tanks."

Is the above statement due to the fact that there are paradoxes in nature or that engineers have not found the answers to occult factors inherent in the hydro-mechanical workings of sedimentation? No doubt on account of this quandary engineers have used rule-of-thumb methods for too long a time in designing such works.

In the last paragraph of the author's summary, it is suggested that further research and experimentation be conducted with a view toward designing rectangular tanks. This is well taken and should be welcomed by all sanitary engineers.

The average practicing engineer would appreciate having a few safe rules

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3. Heffer states in the Manual of British Water Supply Practice, page 360, 1950 Edition.
4. Public Health Engineering, Vol. 1, page 440.

or guiding parameters for designing water treatment settling basins. In this regard the writer studied this problem with a glass model on 1:20 scale and using potassium permanganate dye tracers which could be photographed through the sides of the glass walls. The problem was to find the economic solution for the design of water treatment tanks which necessarily had a relatively short width to length ratio, since to make long tanks would have required the excavation of a high rocky hill at great cost. The water supply was turbid varying from 60-1100 ppm. The coagulant was to be alum. The precipitated sludge had a water content of 93 per cent. Sludge storage was to be provided for a maximum period of 6 months, which would require a deep tank. Raw water was to be well flocculated prior to entering the tank. The entrance of influent water was to be projected downward from four inlets at average velocities of 1 ft. per sec. A surge baffle which was solid for the upper half of the tank depth was set 10 feet out from the influent end wall. This absorbed the energy of the rebound from the bottom and forced the water downward again so that even horizontal flow occurred. In order to make a greater length-width ratio, a longitudinal septum was set axially as a training wall, and it provided three disconnected sections through the middle of the tank. Each end section was 12 feet long and the central section 9 feet long. These septum walls supported baffles at the influent and mid section, and two rows of overflow weir channels at the effluent section. The mid baffle was designed as a venetian blind with 2" x 14" redwood boards as horizontal slats. The top half and bottom half were designed to operate separately and to be set at any angle of opening or closure with the vertical. The bi-sectional arrangement was required for top and bottom variation, due to the fact that stratification would occur with small temperature differences.

The design as finally adopted was as follows:

Overflow rate: At maximum flow 900 g/sq.ft./D, at average flow 600 g/sq.ft./D.
Detention time with clean tank: Maximum flow 3.6 hrs., at average 5.4 hrs.
Detention time with sludge: Maximum flow 2 hrs., at average 3.0 hrs.
Width-Length ratio: (considering septum wall) 6, (not considering septum) 3.
Maximum depth-length ratio: (with clean tank) 6.1, (with full sludge) 12.
Maximum velocity of flow with clean tank 0.5 feet per minute, with full sludge load 1.0 foot per minute. The maximum flow rate over the end weir was 25,000 gallons per foot of weir per day, average flow rate over weir 16,700 gallons per foot per day.

The results in prototype practice are gratifying. The turbidity removals are from 96-99 per cent. The effluent turbidity is rarely over 4 ppm and is sometimes 1.5 ppm.

The writer would like to see the overflow residual efficiency in general use, but until some standard method is accepted and put into practice by sanitary engineers, the writer suggests the following method of obtaining efficiency curves which could be plotted on average monthly basis from plant records.

Since the coagulant dosage is influenced by and proportional to low values of color and turbidity, the coagulant dosage required for a given per cent removal may be plotted approximately on a straight line with the sum of turbidity and color values. With higher values of color and turbidity the curve is exponential and can be plotted on logarithmic paper approximately as a straight line. Then, in any given settling tanks for water treatment the per cent removal may be plotted with overflow rates as a curve for each separate ratio of $\frac{\text{turbidity + color}}{\text{coagulant}}$ ratio.

After plotting, there will be many such curves of hyperbolic type which should indicate the optimum and economic point to operate the plant. Of course, if the plant is running at full load, there is no choice but to use a higher coagulant ratio to turbidity and color if the per cent removal is to be increased. If this plan were currently adopted, it would be the means of guiding the operator toward greater economy in the use of chemicals, and his efforts at compilation would be well repaid.

Discussion of
"DESIGN OF TREATMENT PLANTS FOR LOW TURBIDITY WATER"

by Roy H. Ritter
(Proc. Paper 591)

H. L. THACKWELL, M. ASCE.[1]—The general considerations of this paper state that "the design features of modern water purification plants for handling low turbidity water are substantially the same as those for handling other types of surface waters." This is generally true but not universally so. Formerly slow sand filters were used to clarify low turbidity waters generally without coagulation. Currently rapid sand filters are being used to strain out turbidity when it is below 10 ppm and when not involved with high color values. No prior coagulation or settling is provided ahead of the filters. The results obtained by these means is economically justified.

Regarding the trend toward mechanically cleaned, horizontal flow, sedimentation basins for low turbidity water, the writer has noticed only one installation of this type in the Western states. Even in high turbidity waters, many plants have found it economical to use the extra depth sludge basins and to empty them once in 4 - 6 months. One large city in California that had employed mechanical drags for continuously or intermittently removing sludge in rectangular settling basins removed them after several years trial and turned the tanks into ordinary settling basins.

There are many installations of upward-flow mechanically operated basins in the Western states. In this regard the author states "upflow tanks with one or two hours detention have been used in plants with low turbidity waters and high degree of color. Activated silica appears to be necessary to form a sufficiently heavy floc or sludge to make this type of plant operate completely satisfactorily at all times." The proprietary type of upward-flow tanks with mechanical mixing and sludge scraping mechanism are being advocated, by their manufacturers, to operate at high overflow rates and low detention time. The reason for such recommendations is largely due to competition in selling. Low turbidity water with a high color requires just as much detention time as high turbidity with low color. Activated silica is helpful with some plants having high overflow rates.

Upward-flow type tanks having no sludge scraping mechanism and receiving well flocculated water at the influent are successfully operating with low turbidity and high colored water. A recent installation at Oregon City, Oregon, designed to treat 15 MGD of Willamette River water has given the following results without the use of activated silica or clays.

The best operating results were obtained in December and January when there were higher turbidities, the average detention time being 2.5 hours and the overflow rate approximately 1100 gals/sq.ft./D. The lower the turbidity and color the lower the chemical efficiency obtained.

1. Cons. Engr., Walnut Creek, Calif.

Monthly Average	1954 Oct.	Nov.	Dec.	1955 Jan.	Feb.	Mar.	Apr.	May
Turbidity raw, ppm	6.8	10	18	21	8	13	13	3.7
Color raw	19.0	25	39	38	30	38	28	16.0
Sum turbidity & color	25.8	35	57	59	38	51	41	19.7
Turbidity settled, ppm	3.0	4	5.9	4.8	4.5	4.6	3.9	2.3
Color settled	5 -	5 -	5 -	5 -	5 -	5 -	5 -	5 -
Sum turbidity & color	8.0	9	10.9	9.8	9.5	9.6	8.9	7.3
Percent removal	69	74.3	80.8	83.4	75.0	81.1	78.3	63.5
Alum, ppm	17.1	18.8	22.2	18.8	17.1	17.1	17.1	12.0
*Coagulation ratio	1.51	1.86	2.58	3.14	2.22	2.98	2.4	1.64
Flow MGD	5.787	9.034	10.88	10.83	11.58	11.48	11.0	9.80
Overflow gals/SF/D	578	903	1088	1083	1158	1148	1100	980
Detention time, hrs.	4.83	3.08	2.56	2.57	2.41	2.43	2.53	2.85
Flow influent	5.787	9.034	10.88	10.83	11.58	11.48	11.00	9.8
Flow effluent	5.374	8.763	10.64	10.60	11.31	11.27	10.82	9.52
Percent backwash	7.1	3.0	2.21	2.12	2.3	1.83	1.6	2.9
Filter rate g/SF/m	1.78	3.0	3.54	3.52	3.75	3.72	3.6	3.2
Turbidity filtered	1.2	0.9	1.2	1.0	1.0	1.0	0.9	0.4
Color	5 -	5 -	5 -	5 -	5 -	5 -	5 -	5 -

* Coagulation ratio equals $\frac{\text{Turbidity ppm + Color value}}{\text{alum ppm}}$. High ratio number indicates chemical efficiency.

The months of January and March show the best results in chemical economy, and the month of April in percent backwash. Optimum overflow rates 1100-1200 gals/sq.ft./D and detention time 2.54 to 2.32 hours.

With a maximum flow of 15 MDD plant capacity, the surface overflow rate is 1500 gals/sq.ft./D and the detention time is 1.86 hrs. The filtration rate is 4.93 gals/sq.ft./M.

Below a turbidity of 8 ppm and a color value of 20, the alum dose remains constant. This may be the threshhold level for coagulants at this plant.

These tanks are to be cleaned once in four months. One tank being out of use for sludge removal for half a day or 1-1/2 days per year or 3 days for two tanks. The cost of this cleaning work is about $75 per year extra over and above the regular operator's salary. The annual amortized cost of straight line collectors and power for mechanical sludge removal far exceeds this amount.

Horizontal tanks can remove turbidity and color more efficiently than the upward-flow type but in order to do so the surface overflow rate will have to be considerably less than is customary with upward-flow tanks. Upward-flow tanks are indicated in water softening, and also when smaller space is available than required for horizontal tanks. Colored water from deep wells is

generally more difficult and costly to remove than colored surface water. With very high color, a high dosage of chlorine in conjunction with ferric sulphate coagulant is required for color removal in conventional type settling basins.

As stated by the author, "In the spring of 1953 with slightly higher than average color, low turbidity and low overflow rate, the alum use was the lowest. This demonstrates that, regardless of color, the low overflow rates permitted a reduction in alum dosage." This was undoubtedly due to the longer detention time obtained, which is more important in color removal than in turbidity removal.

A well designed basin should be able to remove turbidity and medium color values to less than 5 ppm. Excess coagulant will be able to compensate for overloads that have surface overflow rates above 900 gals/sq.ft./D or for turbidity surcharges.

When possible in a given water, the coagulant requirements should be studied with jar tests over a year's time. The designer should then be able to determine peak conditions as well as average conditions and be able to set the average rated plant capacity at the optimum point for maximum economy.

Discussion of
"SAND FILTRATION STUDIED WITH RADIOTRACERS"

by Donald R. Stanley
(Proc. Paper 592)

THOMAS R. CAMP,[1] M. ASCE.—The technique described by the author of studying the mechanism of sand filtration by means of radioactive tracers is a new tool which should be helpful in the development of a rational theory of filtration of water through sand. The author is correct in his statement that despite the long period of development and the present widespread use of rapid sand filters, little fundamental information concerning the mechanism of the removal of suspended matter by filters has been obtained. This is a field which has long been neglected by sanitary engineers and it is deserving of rigorous theoretical and experimental study. The writer has been interested in and has participated in research in this field for about 25 years. The studies by Eliassen and Stein, referred to by the author, were supervised by the writer when he was a member of the faculty of the Massachusetts Institute of Technology. The purpose of this discussion is to comment on some of the statements and conclusions drawn by the author in the hope that the comments may clarify our thinking on the problems.

The concept of depth of "penetration" of floc into a sand filter is unreal and not likely to lead to the development of a rational theory of filtration because some floc always penetrates completely through the filters. Since the sanitary engineer is interested primarily in what passes through the filter, any worthwhile theory must take this into account. In the work done by Eliassen and Stein, the concentration of suspended matter in the water at various depths in the filter was measured and used as a basis for the development of a theory. In the author's work the concentration at various intervals of time of the suspended matter deposited in the bed was measured. Both types of measurements are required for a satisfactory development of a rational theory.

In the writer's early studies of the problems of filtration he had a small tubular glass filter constructed in sections with flanged joints and screens at the end of each section to contain the sand therein. The purpose of this apparatus was to measure the accumulated floc in each section of filter after various intervals of time by weighing the dried section before and after a run. The method proved unsatisfactory because the weight of dried deposits was too small in comparison with the weight of the section containing the deposits. The author's technique with radiotracers solves this problem.

The author's Fig. 3 showing the iron concentration in the filter sand at various depths for various intervals of time should be compared with similar graphs developed from the Eliassen experiments which show the concentration of iron in the water inside the bed at various depths after various intervals of time. One of these graphs was reproduced by the writer in Davis' "Handbook of Applied Hydraulics" 1952, page 982. The shape of the curves is the same but the scale of iron concentration is vastly greater for the author's graph.

1. Camp, Dresser & McKee, Cons. Engrs., Boston, Mass.

For example, 1.0 milligram of iron per cc of sand is equivalent to about 2500 parts per million of iron in the pore water. The iron fraction actually in the pore water and not attached to the sand grains is thus infinitesimally small as compared to the scale of the author's Fig. 3. The author's Fig. 3 would tend to indicate that none of the iron passes through the filter whereas some iron always passes completely through a filter. In this connection, the author's Fig. 2 should be explained. The curves must be asymptotic to the horizontal axis although they are not so drawn.

The author uses the results of his experiments to develop some empirical equations relating the penetration index to the sand size and the rate of flow. These equations do not add to our understanding of the mechanism and they should not be used in practice because they do not account for size and variations in size of the floc and stratification of sand according to size in a rapid sand filter. Moreover they do not include any parameter relating to the time rate of head loss. Since sand size, remaining porosity, floc concentration and floc size vary both with depth and time during a run, the variables must be correlated for a particular depth and time and then integrated over the entire depth and length of run. It is difficult to see how the concept of penetration will be helpful in deriving a rational theory.

The experiments of Eliassen show conclusively that the burden of removal of floc is greatest at the top of the bed and at the beginning of a filter run, but that this burden is gradually transferred to a lower depth in the bed so that near the end of a long run the top sand is removing nothing. In these experiments, coarse sand was used to avoid settlement of floc on top of the bed. During this process of transfer of the burden of removal to lower levels in the bed, the concentration of floc in the effluent increases. Nevertheless, there is some removal in the bottom sand all of the time. Any satisfactory theory must take into account all of these observations.

The author's Fig. 5 indicates that head loss increases directly with the total amount of floc deposited in the bed and hence directly with time for a constant rate of filtration. This observation or an approximation to it has been observed in a great many filtration plants; but on the other hand, if the filter run is long enough, the head loss will increase at a faster rate toward the end of the run. In the writer's studies of porous plates for filter bottoms, about 1934, experiments were conducted on accelerated clogging of the plates by floc. An increase in the rate of increase of head loss near the end of these runs was especially noticeable.

The controversy as to whether sedimentation occurs within a filter bed is not likely to contribute anything of importance to a rational theory. The fact of the matter is that a floc particle is removed from the water by coming in contact with the surface of a sand grain, or the floc already on a sand grain and adhering thereto. It makes little difference whether the particle settles out of the stream line carrying it onto the surface or stays in the stream line and reaches the surface by chance. The important thing is that contact must be made to effect removal of the particles.

Stein's observations under the microscope show that a part of the floc sloughed off of the sand grains on which they were first deposited to pass deeper into the bed. The author presents results of experiments which tend to show that this does not occur. He found that the total net count and the shape of the distribution curve was the same before and after passing distilled water through filters. This indicates that the flow of distilled water did not disturb any appreciable amount of the floc deposited within the bed; but it does not indicate that when floc is passing into the bed, the deposit of a

particle of floc at a particular point may not be accompanied by the dislodging of another particle of floc just downstream therefrom.

It is evident from the experiments of Eliassen, and also indicated by the shape of the curves in the author's Figs. 2 and 3, that when the velocity through a pore becomes too large the shearing force on the walls exceeds the bond strength of floc particles and no further removal occurs. The question as to whether some of the floc particles in the deposits on sand grains shear off and pass deeper into a filter is important in deriving a rational theory. It is evident that the particles which remain in the deposits are strong enough in shear to withstand the shearing forces to which they are subjected; and, if they do not shear off as the shearing forces increase with clogging their adhesive and shearing strength must be greater than any shearing forces which can be introduced by clogging.

Any rational theory for the filtration process must take account of the conservation of the pore space within the bed and the effect of reduction in pore space by clogging on the increase in head loss. Both the author's experiments and the experiments by Eliassen took account of floc concentration in terms of the weight of iron in the floc. It is not the weight of material in the floc, but rather the volume of the particles that occupies the pore space. Hence, some correlation must be made between weight and volume. The weight of iron is only an index to the volume occupied by the deposits, and the ratio of weight to volume may be far from constant.

Stein's studies were an attempt at the formulation of a theory relating the physical factors. In his studies he utilized the Kozeny theory for the flow of fluids through granular materials and found that the equation could be adapted to the bed containing the deposits. The writer believes that Stein's work was a real step forward in the development of a rational theory. Experiments are useful in checking and interpreting theoretical derivations. It is preferable to derive the theory, step by step, with the experimental apparatus adapted for verification purposes.

The author's Fig. 7 shows the effect of the pH value of the suspension on the penetration index. It has long been known that time of formation of floc is dependent upon the pH value and that there is an optimum pH value at which floc forms most rapidly. To the best of the writer's knowledge the author is the first to discover that there is an optimum pH value at which floc filters most effectively. It is too early to express an opinion as to why this is so, but the finding opens many avenues for speculation and future research.

The writer is intrigued by the author's suggestion that filters be operated at high rates at the beginning of a run and that the rate be decreased during the run. Most plants are designed so that the filters should be operated at a constant run throughout each run. This simplifies the problem of addition of the chemicals. The Detroit experiments of about 30 years ago, and more recent experiments by Baylis and others, tend to indicate that uniform filter rates up to 3 gpm per square foot may be used almost anywhere and that on some occasions rates as high as 5 gpm per square foot may be used. With the author's suggestion even higher rates may be used. For example, if during a run a filter starts at a rate of 10 gpm per square foot, and the rate is uniformly decreased to 2 gpm per square foot, the plant production is at an average rate of 6 gpm per square foot. Since the rate was only 2 gpm per square foot at the end of the run the chance for bad effluent quality is small. This type of plant operation could be easily adapted to large filtration plants, but it might be somewhat difficult for small plants with a small number of filters.